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CONTENTS

PAPERS AND DISCUSSIONS

	<i>Page</i>
The Action of Impulsive Loads on Elastic Structures. <i>John B. Wilbur</i>	163
Hydraulic Design of Conduit Intakes in Dams. <i>Byron</i> <i>O. McCoy</i>	174
Clamshell Bucket Unloading Towers. <i>Henry Brask</i>	185

OF GENERAL INTEREST

Proceedings of the Society	200
--------------------------------------	-----

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JOURNAL OF THE
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Volume XXXIII

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**THE ACTION OF IMPULSIVE LOADS
ON ELASTIC STRUCTURES**

BY JOHN B. WILBUR, MEMBER*

(Presented at a meeting of the Boston Society of Civil Engineers held on March 20, 1946)

INTRODUCTION

To resist satisfactorily a bomb by which it is struck, a roof slab should have sufficient thickness to resist penetration or harmful scabbing. For this discussion it will be assumed that such thicknesses have been provided. Our attention will be devoted to methods of analyzing a structure for the bending moments, shears and direct stresses that are set up in the structure as the result of the application of impulsive loads.

IMPULSIVE LOADS

An impulsive load is one which varies in intensity with time. The application of wind pressure to a skyscraper constitutes such a load. In connection with bombs there are three important types of impulsive loads: (1) Blast loads resulting from the explosion of a bomb in air (including explosions in contact with a structure); (2) Impact loads resulting from the blow delivered to a structure by a bomb of a given mass and velocity; and (3) Ground shock, due to the explosion of a bomb in the earth at some distance from the structure.

Because the time element is involved in dealing with impulsive loads, the methods necessary to analyze a structure for the resulting

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moments and shears are essentially different from those that would be used for static loads. In the design of bridges, the determination of impact stresses poses a similar problem, but impact stresses in bridges constitute such a small proportion of total stresses, that relatively crude methods suffice for their determination. The impulsive loads caused by bombs, however, lead to a major portion of the stresses to which a structure may be subjected. This requires a careful consideration of the analysis of structures acted upon by impulsive loads. It is worthy of mention that these methods, developed for the purpose of investigating the effects of bombs on structures, provide expedients for more careful investigations of the effects of wind loads, earthquake motions, impact forces, etc. on bridges and buildings.

VIBRATIONS OF ELASTIC BODIES

To investigate the action of impulsive loads on elastic structures, it is necessary to consider the elastic vibrations of the structure.

A structure is said to have one degree of freedom if its state at any instant can be defined by one number. Fig. (1) illustrates

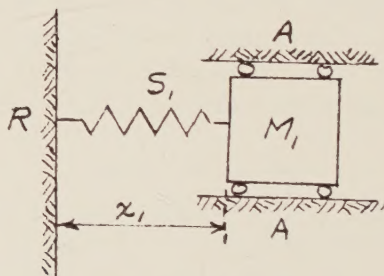


FIG. 1

such a system, since the position of the mass M_1 can be defined at any instant by the dimension x_1 . Fig. (2) shows a system with two degrees of freedom, in which the position of masses M_1 and M_2 are defined at any instant by the two dimensions x_1 and x_2 . An elastic structure, such as a beam, has an infinite number of degrees of freedom, because the definition of its position requires the enumeration of the ordinate of the deflection curve for every point on the beam, and there are an infinite number of such points.

There are as many normal modes of vibration for an elastic

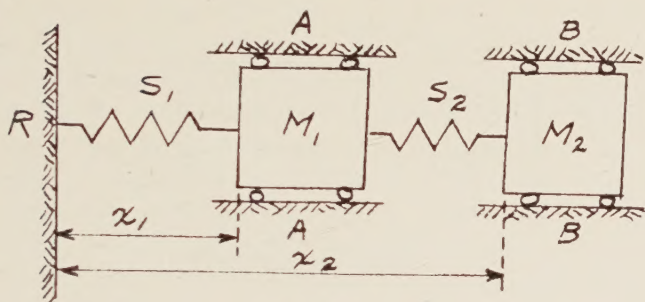


FIG. 2

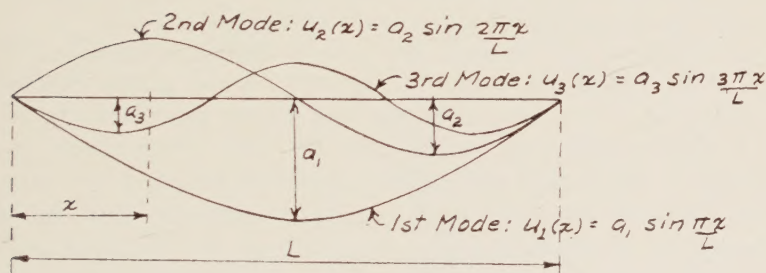


FIG. 3

structure as there are degrees of freedom. Thus the structures of Figs. (1), (2) and (3) can vibrate in one, two and an infinite number of modes respectively. With each normal mode of vibration, there are associated, first, a period, i.e., the time required for one complete cycle of vibration, and second, a characteristic shape i.e., a function that defines the geometrical pattern in which the vibration occurs.

SYSTEM WITH ONE DEGREE OF FREEDOM

Fig. (4) shows a system of one degree of freedom, acted upon by an impulsive load P , which varies in magnitude with time. Because of the spring S the mass M can and will move horizontally. At any instant the mass M is acted upon by three forces, all of which vary with time: (1) The impulsive load P ; (2) The tension or compression in the spring S ; and (3) The inertia force resulting from the acceleration of the mass M . The vector sum of these three forces must at all times be zero, and this fact permits the writing of a differential equation, the solution of which gives the displacement x

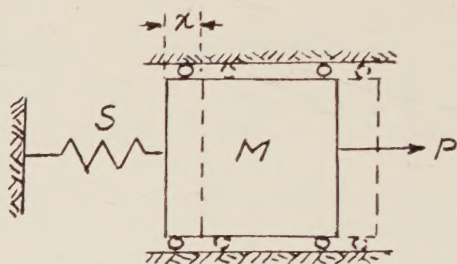


FIG. 4

at any time t . Since this displacement equals the strain in the spring, the stress in the spring at any time t is thereby defined.

It is well known that if the load is applied gradually and remains at its maximum value P , the stress in the spring becomes P and maintains that value. It is also well known that if the full value of P is applied suddenly, and if the full value P continues to act, the stress in the spring will oscillate between zero and $2P$. For the general case of impulsive loads, however, recourse must be made to an analysis as outlined above. But without such detailed analysis, certain observations can be drawn. Suppose the load P were applied suddenly, but remained in action over a certain period only. If this period of application were extremely short, the mass M would not be moved through a distance sufficiently large to cause large stresses in the spring. The importance of time and mass effects becomes apparent, and it is seen that extremely large intensities of impulsive loads may, under certain conditions, cause only moderate stresses.

Fig. (5) illustrates a typical impulsive load. If this load is applied to a system with one degree of freedom, such as the structure of Fig. (4), the stress in the spring will vary with time. At any

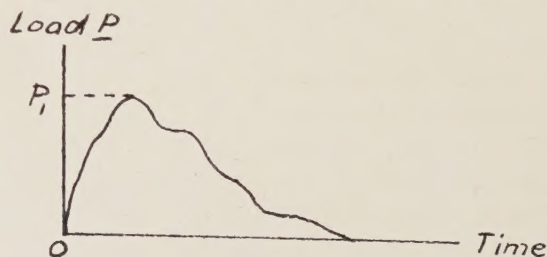


FIG. 5

instant after the initial application of the load, let the stress in the spring be defined as $D_t P_1$, where P_1 is the maximum value of the impulsive load, and D_t is called the dynamic load factor. The dynamic load factor corresponds to the impact factor. The value of D_t at any time t can be determined, for any given impulsive load acting on any given system of one degree of freedom, by the differential equation analysis previously described. Values of D_t could be referred to any of the various ordinates to the curve representing the impulsive load, but for convenience it is arbitrarily referred to the maximum ordinate P_1 .

Fig. (6) shows an impulsive load of a special form, namely, a

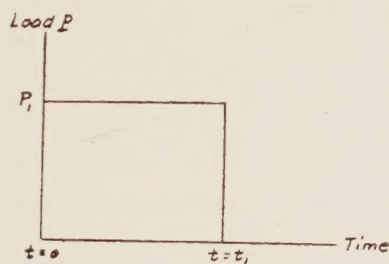


FIG. 6

rectangular pulse, wherein the load P is applied suddenly and maintains its maximum value P_1 during the time interval t_1 . In Fig. (7) values of dynamic load factors for a rectangular pulse are plotted against a time scale. To make the curves applicable to all structures with one degree of freedom, the time scale used is t/T , where t is the time elapsed since the initial application of the rectangular pulse, and T is the period of vibration of the structure. It will be noted that the maximum positive and negative values of D_t vary appreciably with the ratio t_1/T . Thus for $t_1/T = 1/2$ (i.e., the duration of the pulse equals half the period of vibration) D_t oscillates between $+2$ and -2 . However, as t_1 becomes smaller relative to T , the maximum values of D_t are reduced. For $t_1/T = 1/6$, for example, D_t varies from $+1.0$ to -1.0 . For still smaller values of t_1/T , the maximum values of D_t would be still further reduced. The effect of a large value of P_1 , applied through an extremely small time t_1 might lead to very small stresses. By an extremely small time, one obviously refers to time relative to the period of vibration of the structure.

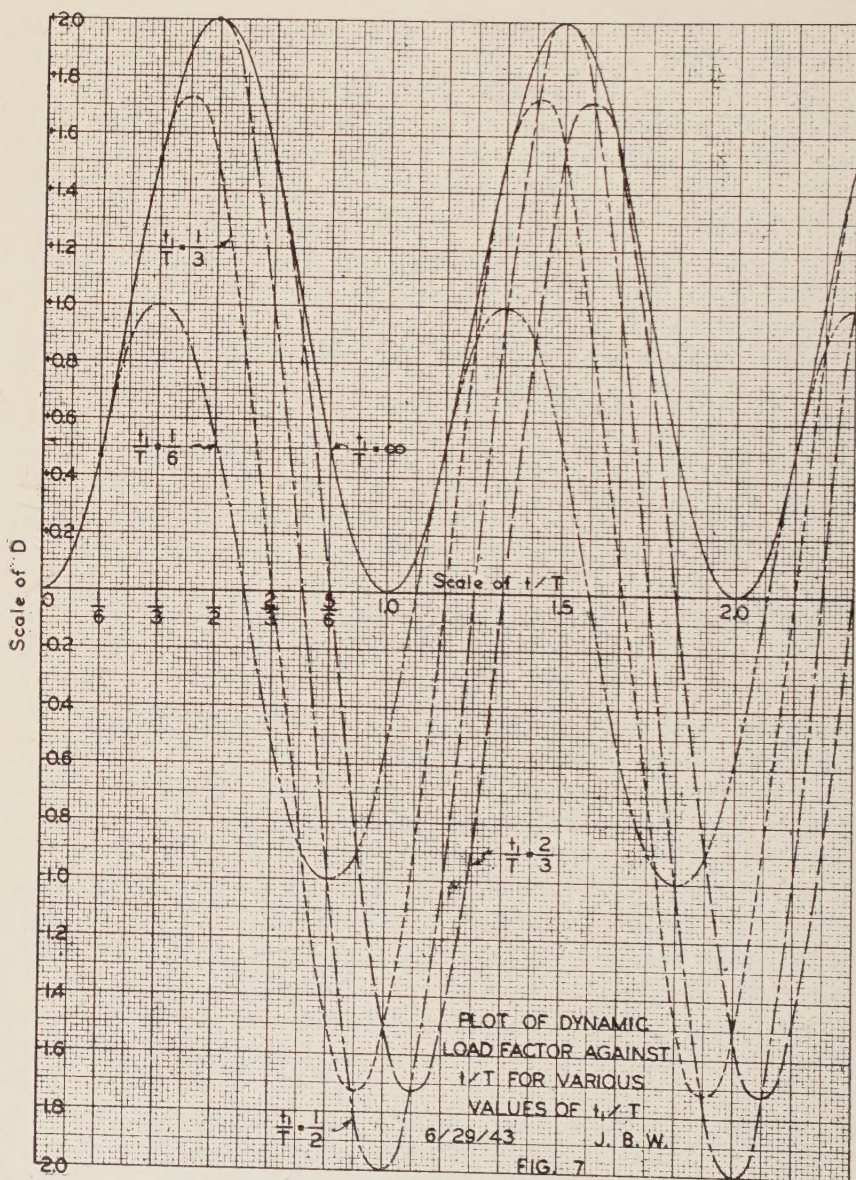


FIG. 7

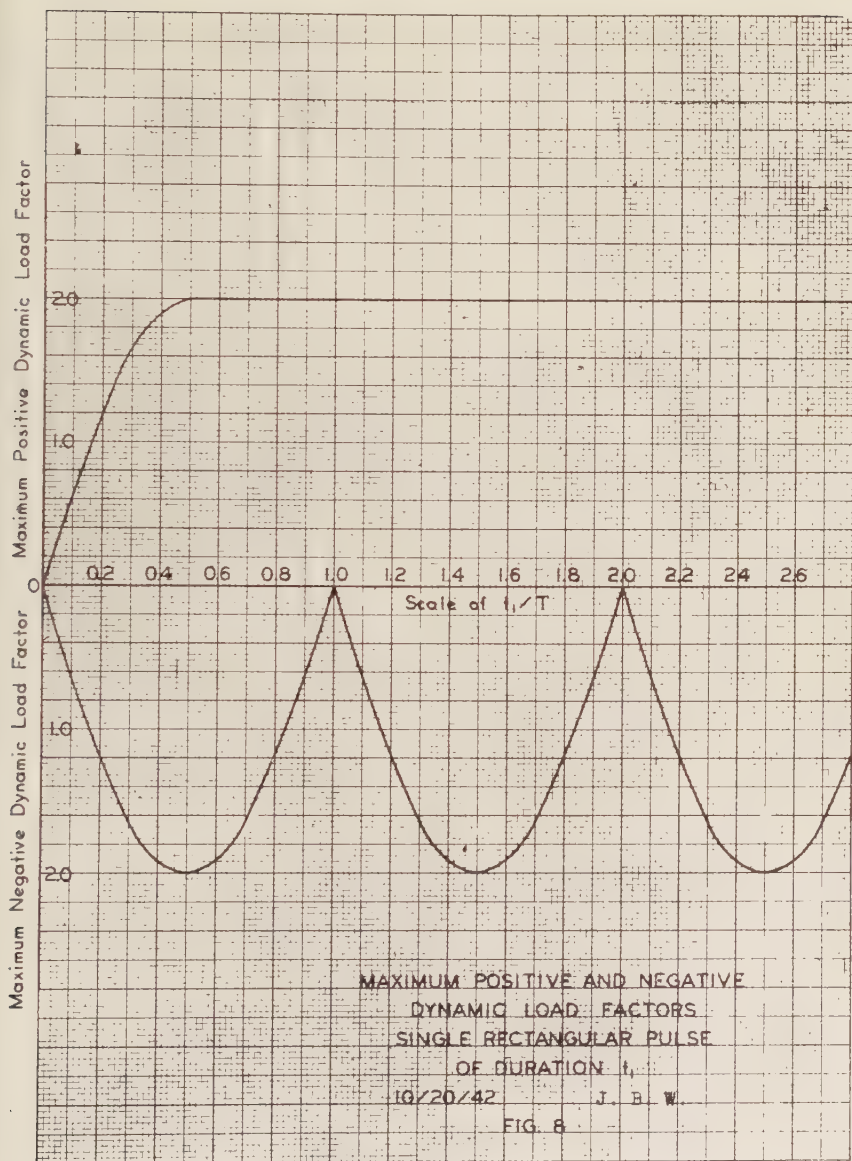


FIG. 8

Since this period depends on the inertia and elastic properties of the structure, the determination of stresses in the spring must include an evaluation of the effects of these parameters.

For design purposes, we are interested in maximum stresses, which correspond to maximum values of dynamic load factors. For the rectangular pulse, such values are plotted in Fig. (8). Again it is to be noted that these maximum values depend on the ratio of t_1 , the duration of the pulse, to T , the period of the structure. For small values of t_1 , T , the maximum dynamic effect of the impulsive load is substantially reduced.

SYSTEMS WITH MORE THAN ONE DEGREE OF FREEDOM

The extension of the ideas previously outlined, to the analysis of a structure with more than one degree of freedom, is entirely feasible, but is too involved to permit detailed discussion in this brief paper.

Briefly, the spacial distribution of the impulsive load is broken down into what may be called "mode loads." There is one mode load for each degree of freedom of the structure, and each mode load has a spacial distribution that is similar to the characteristic shape for the vibrational mode corresponding to that degree of freedom. If all these mode loads are superimposed on each other, they result in the total spacial distribution of the impulsive load.

Each mode load excites only the mode of vibration having a similar shape, and its dynamic effect may be determined by the use of dynamic load factors in precisely the same manner that would be followed for a structure with only one degree of freedom. The dynamic effect of each mode load is determined separately. Then the effects of all mode loads are superimposed to give the total effect of the impulsive load on the structure.

If a structure has a finite number of degrees of freedom, the effect of each mode must be considered, but actually, higher modes (i.e., modes with shorter periods) may be found to be of negligible importance. For a structure such as a beam, that has an infinite number of degrees of freedom, the results are in terms of an infinite series, which may however converge rapidly.

SIMPLIFICATIONS

For a system with many degrees of freedom the foregoing procedure, while possible, is too lengthy to be of practical value except as a research tool. Important short cuts may be obtained, however, by steps such as the following. Distributed masses may be considered as concentrated at a small number of points, so that one needs to investigate a structure with only a limited number of degrees of freedom. In determining characteristic shapes and periods of vibration, approximate methods of analysis may be employed. Under some conditions model studies may be used advantageously for the foregoing purposes. In combining the effects of the various modes, it can be assumed that the most detrimental effect of each mode occurs simultaneously. Finally, in analyzing continuous structures, considerable liberty is permissible in considering only a portion of the total structure in the analysis.

ACTUAL APPLICATIONS

Using such simplifications the general method as outlined was applied to the design of a number of military structures for the Fortification Division of the U. S. Corps of Engineers. Among the structures analyzed there were included roofs of batteries of various sizes, walls of gun emplacements, blast walls, surface type splinter-proof shelters, foundations for gun blocks, and doors and other closure plates that were exposed to blast and impact loads.

SOME OBSERVATIONS

Because the analysis of elastic structures acted upon by impulsive loads differs in an important manner from an analysis for static loads, one might expect that certain results would suggest ideas of considerable novelty and interest. Such is the case, as will be illustrated by two examples.

We are accustomed to thinking that a beam of given section modulus is more resistant to load for short spans than for long spans. Under impulsive loadings such may not be the case. The longer span leads to longer periods of vibration, so that for a given duration of load, t_1/T is smaller. This results in smaller maximum dynamic load factors. Hence what might be called the equivalent static load becomes less, and this more than compensates for the effect of in-

creased span in computing moments. Of course the beam must carry dead loads also, the effects of which does increase with span. This leads to the idea of an "optimum span", for which a beam of given section modulus can most efficiently resist combined impulsive and dead loads.

We are also accustomed to thinking of moments in a simply supported beam as being greater, for a given load, than they would be for a fixed end beam. For impulsive loads, this may not be the case. The fixed end beam being stiffer, has smaller periods of vibration. This leads to larger dynamic load factors, which result in larger equivalent static loads. Even with the smaller moment factors, the resultant moment in the fixed end beam may exceed the moment that would be caused by the same impulsive load on a simply supported beam. As a matter of fact, little difference will be found between the computed moments for the two cases. This is an important fact, since it permits one to analyze a span of continuous beam (which is actually partially restrained at each end) as if it were end supported.

ELASTIC VS. PLASTIC ANALYSIS

It is to be emphasized that the methods that have been discussed assume that the structure behaves elastically. Actually in the design of a fortification or bomb shelter, it is not economical to carry out a design so that this condition will prevail. Under the bombs for which the structure is designed, the stresses will go well into the plastic range, and should be just short of the stresses that would cause failure.

Under these conditions, an elastic analysis is subject to valid criticism, but the use of elastic analyses for structures wherein the elastic limit is exceeded has considerable precedent. Such a procedure is often followed in aircraft design. Computed stresses that corresponds to the action of the structure after the elastic limit has been reached do not represent true fiber stresses. They are fictitious, but if they can be correlated with the actual failure of structures, they can be used to predict failure. Such a correlation was made, and fictitious ultimate fiber stresses were used in the design problems undertaken.

A plastic analysis would be more rational, and extended studies

have been devoted to the plastic analysis of structures acted upon by impulsive loads. The writer has yet to be convinced that procedures for plastic analysis have yet reached a point where they are simple enough and at the same time general enough in their application to properly supersede elastic analysis. It should be stated, however, that there is disagreement on this point, and that several competent investigators hold to the opposite viewpoint.

One thing is certain. Structures designed to resist bombs must in general be designed on the basis of ultimate strengths. Whether this can best be accomplished on the basis of elastic analyses and fictitious fiber stresses, or by plastic analyses and actual fiber stresses, a new and important problem must receive the attention of the Civil Engineering profession.

ACKNOWLEDGMENT

This paper is based on portions of an investigation conducted at the Massachusetts Institute of Technology for the Office of Scientific Research and Development, and has been cleared by both the Review Branch, War Department Bureau of Public Relations and the Publication Board, Office of the Chief of Naval Operations, of the Navy Department. The discussion of dynamic load factors draws upon Report No. 481 of the David W. Taylor Model Basin of the United States Navy, dated April, 1942, entitled "Effects of Impact on Simple Elastic Structures", by J. M. Frankland. Associated with the writer during the investigation at the Institute were Professor Walter M. Fife, Dr. Charles H. Norris and Mr. Charles F. Peck, all of the Department of Civil and Sanitary Engineering.

HYDRAULIC DESIGN OF CONDUIT INTAKES IN DAMS

BY BYRON O. MCCOY*

(Presented at a meeting of the Hydraulics Section, Boston Society of Civil Engineers,
held on May 22, 1946)

PURPOSE OF DESIGN

From a design standpoint conduit intakes in dams can be divided into two groups, i.e., high velocity and low velocity intakes.

High velocity intakes are those in which water is flowing at or near spouting velocity, with a loss at the intake of most of the head available to cause flow. Under these conditions flow disturbances may cause negative pressure, and cavitation may occur with its resulting pitting effect. The designer is, therefore, particularly interested in providing the proper shape at the intake to avoid cavitation. An excellent paper on the subject of "Cavitation in Outlet Conduits of High Dams" by Harold A. Thomas and Emil P. Schuleen has been published in the Transactions of the American Society of Civil Engineers (1942, Vol. 107, p. 421).

Low velocity intakes are those in which the flow is controlled, and the velocities are a small percentage of the spouting velocity. The control may consist of a valve, a turbine, a restriction in the conduit or merely a length of conduit sufficient to cause a considerable loss of head. The resulting hydraulic gradient provides a positive pressure at the intake and there is little danger of cavitation. Under these conditions the designer is interested in providing the proper shape at the intake to avoid excessive turbulence and resulting head losses.

It is the latter group that will be considered in this paper.

The most straightforward intake would be simply to start the conduit at the face of the dam, as shown in Fig. 1(a). This would give the simplest and cheapest construction and would require the smallest and cheapest gate. However, for the usual conduit sizes and velocities the head loss at the entrance would be considerable. Wherever head is valuable, as in a power development, the head loss is usually sufficient to warrant expenditure to reduce the loss.

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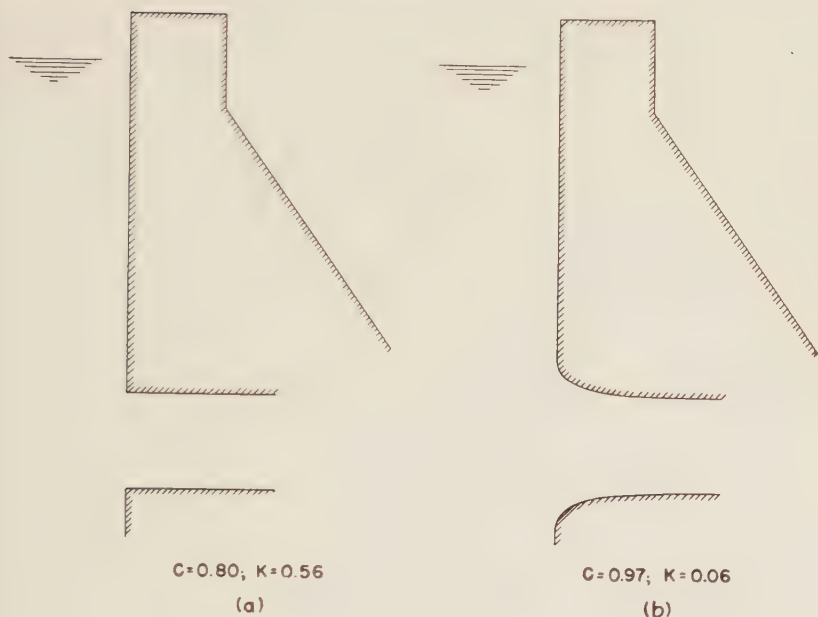


FIG. 1

LOSSES

The losses at conduit entrances are primarily a result of turbulence caused by the changing of direction of the flow as it moves from the tranquillity of the reservoir into the relatively fast flowing stream in the conduit. These are of the nature of friction losses, and shock losses in which energy is lost in the shock of colliding bodies of water traveling in different directions.

The losses at simple conduit entrances have been well established by experiments. In the case of Fig. 1(a) the coefficient of discharge would correspond to that of a tube with sharp cornered entrance and would amount to about 0.80. If the entrance were rounded as in Fig. 1(b) the coefficient would increase to about 0.97.

The head loss at the conduit entrance in terms of the velocity head may be computed from the coefficient of discharge. Since the head available to cause flow is utilized to create velocity head and to overcome losses at the entrance, the formula for discharge can be written:

$$Q = AV = AC \sqrt{2g (h_v + h_l)}$$

in which Q is the conduit discharge, in cubic feet per second, A , the area of the conduit, in square feet, V , the average velocity in the conduit, in feet per second, C , the coefficient of discharge, g , the acceleration due to gravity in feet per second per second, h_v , velocity head, in feet, and h_l , head loss in feet.

Squaring and reducing, the formula becomes:

$$V^2 = C^2 \ 2g(h_v + h_l)$$

$$\text{or } \frac{V^2}{2g} \times \frac{1}{C^2} = h_v + h_l$$

but since $h_v = \frac{V^2}{2g}$ the equation becomes:

$$h_l = \frac{V^2}{2g} \left(\frac{1}{C^2} - 1 \right)$$

Substituting the coefficients $C = 0.80$ and $C = 0.97$ in this equation, the losses are $0.56 \ V^2/2g$ for sharp cornered entrances and $0.06 \ V^2/2g$ for the rounded entrance. Thus, a loss of head equal to one-half of the velocity head can be saved by providing the proper shape at the conduit entrance.

With a flare provided at the entrance, it is economical to locate the gate downstream from the entrance, at the point where the minimum section is reached, where a gate of the minimum dimensions can be installed. However, the installation of the gate introduces additional losses due to the turbulence caused by the gate slots.

These losses are of the nature of shock losses, proportional to the square of the velocity, and are usually quite small. Computations based on test results for the Norris Dam power plant of the TVA indicate that the loss at the penstock entrance including the loss at the gate slot is about 0.2 of the velocity head. Since the entrance to the penstock was carefully shaped, we can conclude that about half the loss is due to turbulence at the gate slots, or about 0.1 of the velocity head.

VALUE OF HEAD LOSS

The power loss due to the loss of head is made up of loss of kilowatts of capacity and loss of kilowatt hours of energy. However, in many instances, the loss of kilowatt hours of energy is the only

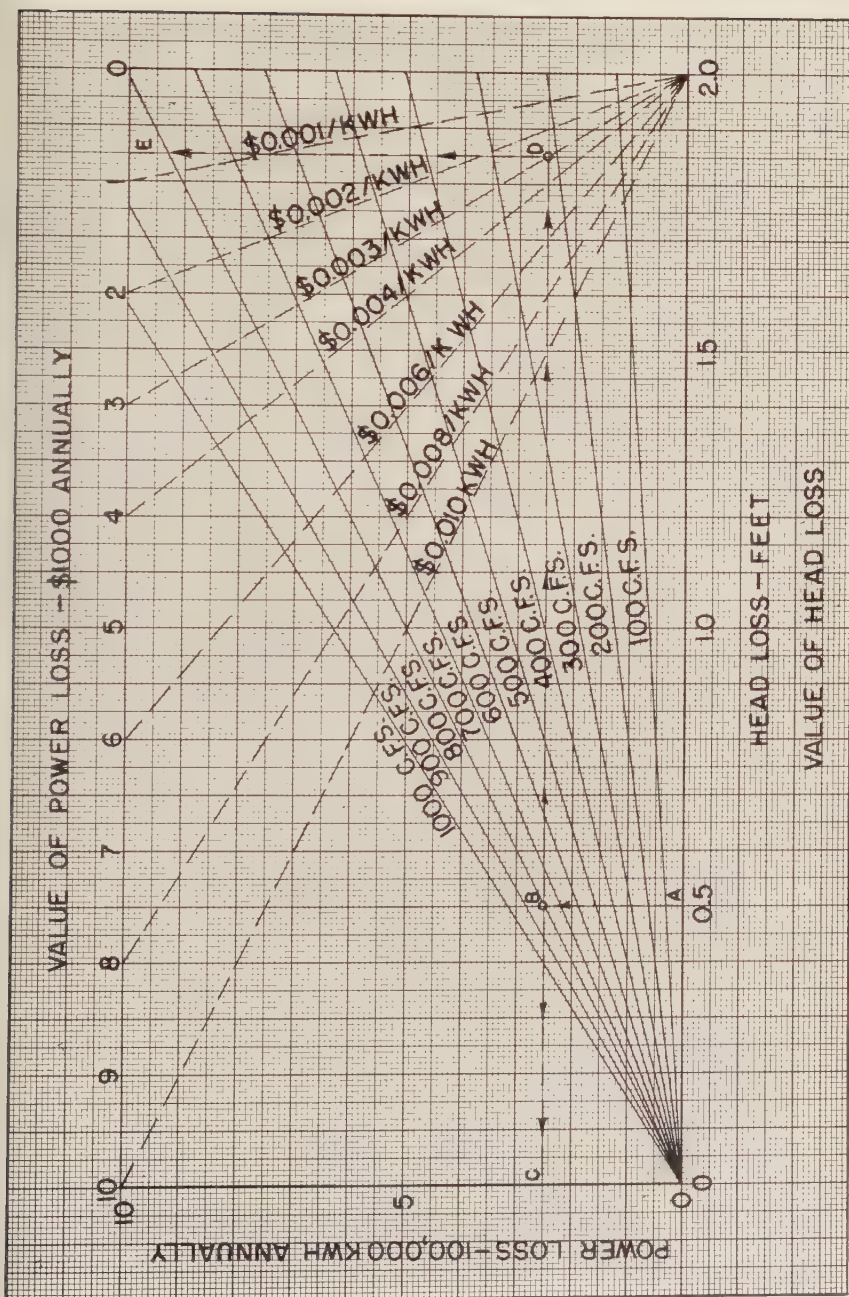


FIG. 2

loss of value to be considered, since the increase in capacity by the saving of head may be too small to make any difference in the capacity value arrived at in contract negotiations for the power, while the energy saving will actually be accounted for by metering.

The chart in Fig. 2 gives the loss in kilowatt hours, and their value, for various head losses and flows.

For a velocity of 8 ft. per second, a typical figure for power penstocks, the velocity head is about 1 ft. The saving in head loss due to the proper shaping of the entrance would then be about 0.5 ft. For a flow of, say, 800 cfs, the kilowatt hour loss is seen to be about 250,000 kw hours per year. With kilowatt hours valued at 3.0 mills per kw hour, the annual loss would amount to about \$750. With the annual cost of additional investment amounting to about 10 per cent of the investment, we could afford to invest initially up to \$7,500 to make the saving. This would pay for a good many square feet of expensive form work at the entrance.

SHAPE OF INTAKE

As already discussed under losses, it is worthwhile to flare the entrance to the conduit. The best shape of entrance for either a high velocity or low velocity intake is that conforming to the shape of a jet from a sharp edged orifice.

The shape of the jet is a function of the dimensions and geometrical configuration of the entrance, irrespective of the head. The jet from a narrow slit has been found to reduce to 0.62 of the slit opening, while the jet from a circular orifice reduces to a diameter of 0.77 of the diameter of the orifice.¹ The reduction in the width of a jet from a rectangular orifice would depend on the ratio of the sides of the rectangle. For a square orifice the reduction of the jet width would be only slightly greater than that for a circular orifice. However, as one side of the rectangle becomes greater in relation to the other side, the reduction of the jet width increases, approaching the reduction of the jet from a narrow slit.

Fig. 3 gives the coordinates for the shape of an ideal entrance conforming to the shape of a free jet from a narrow slit. These coordinates will be on the safe side for any other shape of orifice.

¹"Cavitation in Outlet Conduits of High Dams", Trans. Am. Soc. C.E., Vol. 107 (1942), p. 421.

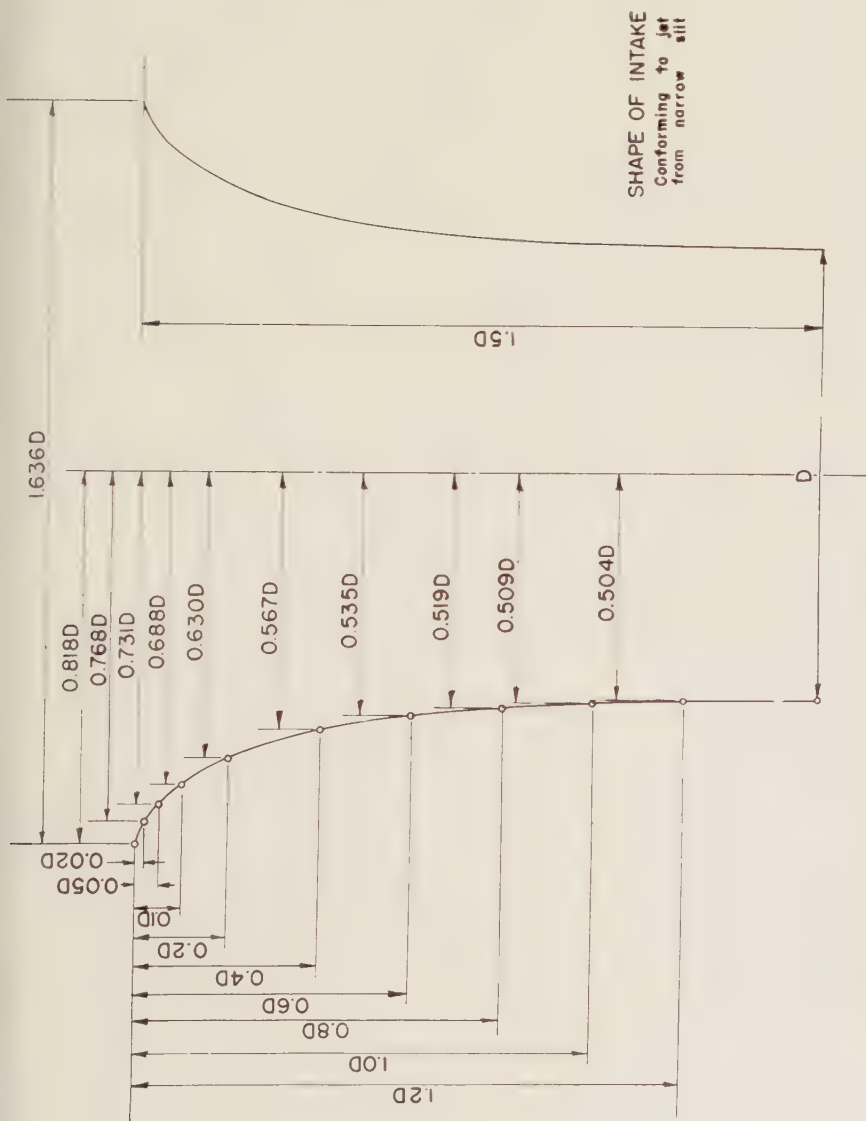


FIG. 3

The shape of the jet, as shown in Fig. 3, has been derived theoretically by Kirchhoff and conforms to the following equation:²

$$Y = \sqrt{2X - X^2} - \frac{1}{2} \log_e \frac{1 + \sqrt{2X - X^2}}{1 - \sqrt{2X - X^2}}$$

with the "Y" axis parallel to the axis of the conduit, and the origin at the edge of the orifice at the entrance.

Others³ have suggested that the intake conform to an ellipse with semi-major axis equal to the distance from the orifice to the "vena contracta", and the semi-minor axis equal to one-half the difference between the diameter of the orifice and the diameter of the jet at the "vena contracta".

In the case of the jet from a narrow slit, the "vena contracta" is taken at a distance from the slit, approximately equal to the jet thickness at the "vena contracta". The jet thickness at the "vena contracta" is taken equal to 0.62 times the slit opening. The resulting elliptical equation is:⁴

$$\frac{X^2}{D^2} + \frac{Y^2}{(0.310D)^2} = 1$$

or, $X^2 + 10.4 Y^2 = D^2$

Another elliptical equation which has been used is:

$$\frac{X^2}{D^2} + \frac{Y^2}{(D/3)^2} = 1$$

or, $X^2 + 9Y^2 = D^2$

A comparison of the shape of these equations with that of the narrow slit jet is shown in Fig. 4.

The Bureau of Reclamation found⁵ from model and prototype tests of the 102 in. conduits through Grand Coulee Dam, that an elliptically shaped profile for the sides of a circular entrance gave good results with respect to cavitation conforming to the following equation:

$$\frac{X^2}{(0.5013D)^2} + \frac{Y^2}{(0.15041D)^2} = 1$$

²This equation was derived for this paper by Harold A. Thomas, Jr., from the analysis by Lamb in "Hydrodynamics".

³"Cavitation in Outlet Conduits of High Dams", Trans. Am. Soc. C.E., Vol. 107 (1942) p. 421.

⁴See footnote one.

⁵"Handbook of Applied Hydraulics"—C. V. Davis, p. 398, McGraw-Hill, N. Y. 1942.

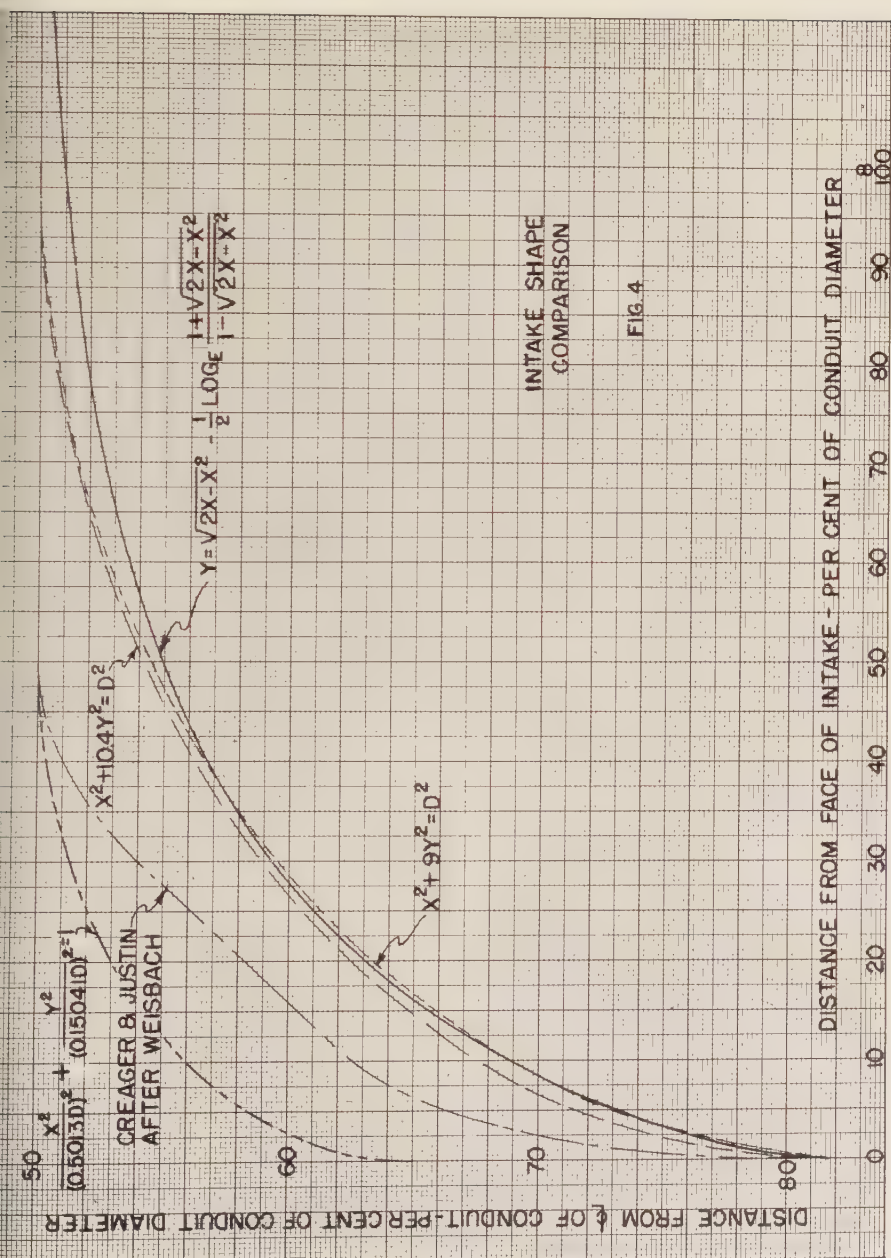


FIG. 4

This equation has been plotted in Fig. 4. A comparison of the "bell-mouth" shape suggested by Creager,⁶ after Weisbach, conforming to the jet from a circular orifice is also shown in Fig. 4.

TRASH RACKS

In the case of penstocks it is necessary to have trash racks at the conduit entrance in order to prevent debris from entering and damaging the water wheel. In some plants, where hand raking of the trash racks is involved, the velocities at the racks are kept down to 1.5 to 2.0 ft. per second to facilitate raking, on plants where mechanical raking is used, higher velocities at the racks can be permitted up to 2.5 to 3.0 ft. per second. In large plants with large reservoirs and wide rack bar spacing or plants with deep intakes, there will be little if any need for raking, and higher velocities at the racks can be permitted. Velocities as high as 5 ft. per second have been used.⁷

Usually these rack velocities are lower than the velocities would be at the entrance of the ideal bell-mouth. It is therefore necessary to provide a greater flare between the trash racks and the penstock. This can often be done by merely increasing the flare of the roof. Where this is insufficient the flares of the other three sides can be increased. The increased flare should have no curvature sharper than that of the ideal bell-mouth. This can best be achieved by increasing the dimensions of the flare in direct proportion to the increase in flare desired.

From a structural standpoint it is desirable to have rectangular racks and gates, particularly for large conduits. However, it is usually desirable to have a circular conduit from both a structural and hydraulic standpoint. Therefore, a rectangular cross-section is maintained throughout the flare from the trash racks to the gate, with a transition from rectangular to circular in a distance of about one conduit diameter beyond the gate.

GATE SIZE

For very large conduits the gates become quite large. There are several advantages to using a single large gate instead of dividing the entrance into two or more openings with smaller gates.

⁶"Hydro-electric Handbook" by Creager and Justin, p. 114, John Wiley & Sons, N. Y. 1927.

⁷"Trend in Hydraulic Turbine Practice" by I. A. Winter, Trans. Am. Soc. C.E., Vol. 106 (1941) p. 331.

From the standpoint of hydraulics the splitting up of the entrance is objectionable on several grounds. In the first place, for the same total area, the wetted perimeter is increased 50% or more depending on the number and relative proportions of cross-sections involved. In the second place, no matter how carefully the piers between the gates are shaped, they will increase the turbulence of the flow into the penstock. A vortex trail will be set up at the downstream end of the pier which will cause violent turbulence in the penstock. In the relatively short penstocks usually involved, such turbulence would carry into the scroll case of the turbine and is bound to reduce its efficiency somewhat. With a reduced gate area the turbulence effect of the gate slots is increased. While these hydraulic disturbances cannot be evaluated quantitatively, they are of sufficient importance to warrant making every effort possible to eliminate the piers and use a carefully shaped single gate entrance.

From a structural standpoint, the single gate is preferable until it attains a size which is not practicable to handle. It can be said that in general fewer and larger units are cheaper up to limits of shipping and practicability of handling. While the multiple gates are smaller and easier to handle, with shorter spans and lighter structural members for the same total area, the mechanical details of the gates would be increased.

The reduction in loading on these details would be more than offset by the doubling or tripling of the number of units. This includes the hoisting mechanism as well as the rollers or wheels. In addition, two extra sets of guides would be required for each additional gate, involving a considerable quantity of structural steel.

Actually, the difference in cost between a single large gate and two smaller gates is not very great. The principal item of additional expense is the reinforced concrete pier between the gates. This is not necessarily additional concrete because the amount of concrete in the intake is usually determined by stability requirements. If the concrete was not put into a pier it would have to be installed somewhere else to provide mass and prevent the intake section from overturning or sliding. However, in the pier the concrete is relatively expensive, not only because of the additional forms required but because of the added cost of handling and placing the concrete. With a single gate, only mass concrete is involved, as a part of the dam.

Introducing reinforced concrete into the construction procedure rearranges and slows down the whole program of concrete handling and placing. With a construction organization geared to high speed to handle mass concrete for a large dam, the delay in handling and placing the small amounts of concrete for these piers can amount to a considerable sum.

The writer had occasion to investigate this feature recently in connection with the design of two large hydro-electric projects of over 200,000 kw capacity each. The cost of these piers was estimated to be over \$100,000 for each project.

From the standpoint of practicability we have probably not yet built the largest intake gate possible. The following table lists some of the largest intake gates which have been built to date:

<i>Name of Plant</i>	<i>Date of Installation</i>	<i>Penstock No. of Gates</i>		<i>Size of Gate</i>
		<i>Diameter</i>	<i>Per Penstock</i>	
Marshall Ford	1939±	16 ft.	1	16 ft. x 22.5 ft.
Hiwassee	1939	18 ft.	1	14.67 ft. x 25 ft.
Grand Coulee	1941	18 ft.	1	15 ft. x 29.65 ft.
Cherokee	1941	19 ft.	1	14.5 ft. x 23.5 ft.
Norris	1935	20 ft.	1	16.5 ft. x 28.5 ft.
Wolf Creek	1946	20 ft.	1	17 ft. x 28 ft.
Center Hill	1946	20 ft.	1	17 ft. x 28 ft.
Norwood	1927±	23 ft.	1	26 ft. x 23.5 ft.
Conowingo	1926	27 ft.	2	22 ft. x 44 ft.
Spier Falls	1930	26 ft.	4	12 ft. x 17.5 ft.
Shipshaw	1943	30 ft.	2	20 ft. x 31 ft.

CLAMSHELL BUCKET UNLOADING TOWERS

BY HENRY BRASK, Member*

(Presented at a meeting of the Designers Section, B.S.C.E., held on February 13, 1946)

BULK materials such as bituminous coal, sulphur, phosphate rock, gypsum, and fertilizer materials are usually transported by steamer to some unloading point at a seaport where this material has to be transferred from the ship to storage piles, railroad cars, or to processing plants. One of the most efficient means developed for this transfer of cargo is the Clamshell Bucket Unloading Tower, which digs the material out of the ship's holds and delivers it to a receiving hopper within the tower from which the material is usually distributed by means of belt conveyors. Where the quantity of material to be handled justifies the cost, these towers are made to travel the full length of the ship. When this is the case, the ship is tied up at the pier and the tower propells itself from hold to hold until the vessel is completely unloaded. In some cases where only a limited number of cargoes are received in any year, the cost of such an installation is excessive and a fixed tower is used. In this case the steamer has to be shifted so the tower can have access to the various holds. This shifting of the steamer is accomplished either by the ship's winches or by winches on the pier.

On the North Atlantic Coast the most important bulk material handled in this manner is bituminous coal. In the Baltimore area a great deal of phosphate rock, sulphur, and fertilizer material is transferred from ship to shore by this means. The problem is essentially the same for one bulk material as another, the principal difference being in the unit weight of the substance handled. Bituminous coal averages around 55 lbs. per cu. ft. Sulphur and most of the fertilizer materials average in the neighborhood of 90 lbs. per cu. ft.

In the earlier days of bulk material transferred from ship to shore, cargoes were unloaded by shoveling the material into tip buckets which were then raised and swung inshore by the vessel's tackle. The mast and gaff rig was developed later with a capacity of 60 to 80 tons

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per hour in free digging. Many of these rigs are still in use at small waterfront plants unloading from open barbes. An early type of unloading unit was the incline boom tower from which the horizontal boom tower such as is now used was developed by gradual improvement. Many different individuals and firms have been responsible for the developments and improvements which have led to the modern unloading tower.

Whirley cranes are also used extensively in unloading material from barges. In this case the bucket picks up its load from the open barge and dumps the material into a receiving hopper. In this case the boom of the Whirley has to swing through an ark from the barge to the receiving hopper. For this reason neither the mast and gaff rigs nor the Whirley cranes work out successfully in unloading ships which have standing rigging because of interference with the horizontal swing of the boom. In Europe cranes have been used which by a rapid vertical movement of the boom bring the bucket in after having been lifted vertically by letting out rope. As the boom is raised the bucket is kept in a horizontal position while moving into the hopper. These units are called level luffing cranes.

The unloading tower, or as it is sometimes called the steeple tower, is a decided improvement over the older mast and gaff rig and also over the Whirley in that it provides higher capacities and since the motion of the bucket is always in a vertical plane, there is no interference with the ship's rigging.

The bulk of coal which is shipped to the northeastern states is loaded into ships around Norfolk, Virginia, and comes in steamers varying from 4,000 to 10,000 tons capacity. Coal is carried in several holds which generally have two hatches each. The hatches vary in size and are usually of the order of 20' x 30' in the larger steamers.

When the steamer has been tied up at the pier, the hatch covers are raised and unloading begins. In the process of unloading it is necessary in order to reach the various hatches that either the towers travel along the pier or that the ship be moved. Wherever possible it should be so arranged that the towers can be propelled so as to reach the various hatches since this effects a saving in time of discharging. Towers are usually propelled along the wharf at a rate of 40 to 50 ft. per minute.

The most common type of bucket used in towers is the chain

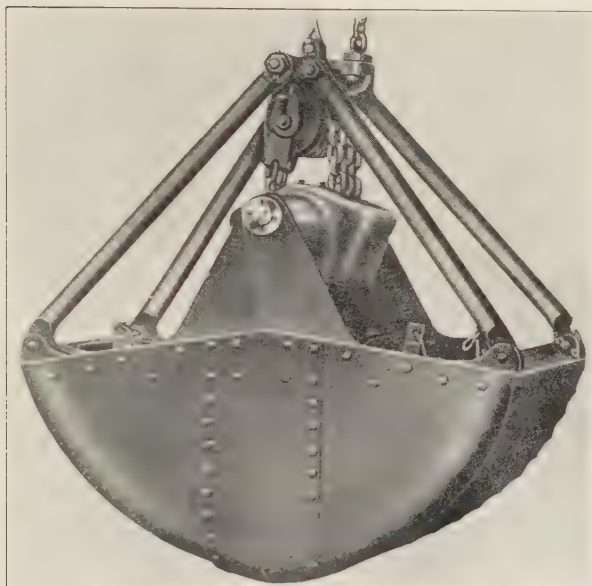


FIG. 1.—CHAIN OPERATED BUCKET
(Robins Conveyors Inc.)

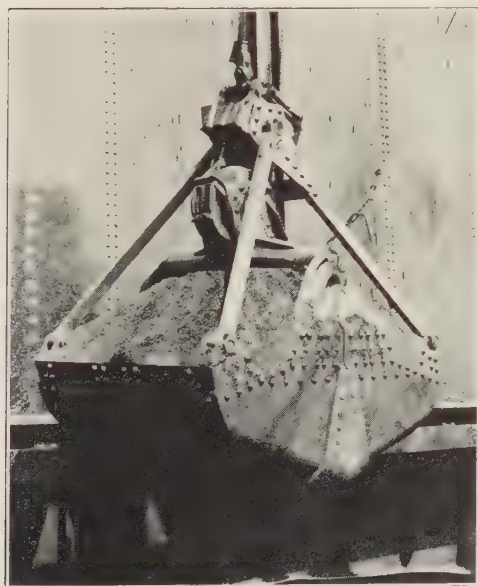


FIG. 2.—“CLYDE RAPID UNLOADER”
(The Clyde Iron Works)

operated grab bucket, as shown in Fig. 1. The closing rope is attached to a chain which closes the bucket. A more recent development is the rope operated bucket, as shown in Fig. 2, in which the closing line extends into the bucket through a system of sheaves.

Tower sizes are generally designated by the nominal capacity of the bucket. These sizes vary from $\frac{3}{4}$ ton capacity in the case of really small plants, up to 4 ton capacity or more. The smallest towers in general use however carry a $1\frac{1}{5}$ ton bucket.

The highest rate of discharge of a cargo occurs at the start of operation in the so-called period of "free digging." This rate is generally referred to as the capacity of the tower, that is the peak capacity. On moderate lifts it is possible for a skillful operator to maintain four or even five complete cycles per minute for a short time. Fig. 3 shows a typical $1\frac{1}{5}$ ton tower which is one of a group of



FIG. 3.— $1\frac{1}{5}$ TON TOWER

three operating on a trestle and depositing coal into cable cars of 4 tons capacity each.

Fig. 4 shows another type of tower in general use which is a steam-operated 3 ton tower traveling on a wharf and delivering coal to a belt conveyor system. Note that this tower has its boiler room below the engine floor.

The peak capacity of a 3 ton tower hauling coal, assuming four trips per minute, would be $3 \times 4 \times 60$ or 720 tons per hour. No operator can maintain this rate of digging for a long period, because he is soon slowed down by reason of the coal getting lower in the



FIG. 4.—STEAM OPERATED 3-TON TOWER

holds and from having to shift from hold to hold; and he is also slowed down by fatigue. When the operator has finally dug out all the coal he can reach with the bucket, trimmers are put into the hold to shovel the coal out to a point where the bucket can reach it. The trimming operation is, of course, the slowest process in discharging a ship. This brings down the average unloading rate to a figure a good deal less than the peak capacity. For example, the peak capacity of a 1 1/5 ton tower would be something of the order of 290 tons per hour. The average through the ship for these towers when operating in a group of two or three is anywhere from 125 to 175 tons per hour each. The average rate through a ship will depend a good deal on the nature of the vessel, the height of the lift, the number of times the ship has to be moved and other factors.

Steam operated towers generally utilize two reciprocating steam engines, the larger of the two being the main hoisting engine with 14 x 24 or 16 x 24 cylinders and having two drums, one for the hold-line and one for the closing line. Superimposed upon this engine is a trolley engine having 10 x 12 or 12 x 15 cylinders and having a single drum for the trolley line.

The trolley engine is usually equipped with a winch head which may be used for swinging the boom and it is sometimes equipped with an arrangement for lifting the boom vertically.

The smaller of the two steam engine sizes given is used for 1 1/5 or 1 1/2 ton buckets and the larger units are used for 2 to 3 ton buckets. Both of these engines are operated by one man who controls both engines through a system of foot levers and hand levers. One foot lever operates the main throttle valve and the two hand levers operate the frictions of the main hoisting drums. One of the hand levers has a latch for controlling the outward travel of the trolley and the other hand lever has a latch for controlling the inward travel. These engines give rope speeds of from 500 to 1000 ft. per minute.

In some cases the trolley engine shaft is extended to provide means of propelling the tower through a system of bevel and worm gearing.

Steam for these towers is provided either from a vertical boiler in the engine room itself or from a central station. Good operation requires 150 to 160 lbs. per sq. in. steam pressure at the cylinders and in sufficient volume to prevent excessive pressure drop. In steam

operated towers other hoist units such as boom hoists, boom swinging units, or independent propelling gear units are usually electrically operated.

In the electrically operated towers, the general arrangement is similar except that there are two separate engines, a main hoist engine and a trolley engine. Both these engines are controlled, however, by one man with a similar system of hand levers and foot pedals. Depending on a rope speed desired, the main hoist is either directly connected to the motor shaft or is driven by the motor through a speed reducer or a set of reduction gears. A separate electrically operated propelling gear for the tower is desirable in the case of heavier towers. This is usually accomplished by a motor propelling the tower through a system of gearing, the most common method being a system of worm gearing driving the wheel axles. Fig. 5 shows a typical general arrangement plan of the engine room of an electrically operated tower.

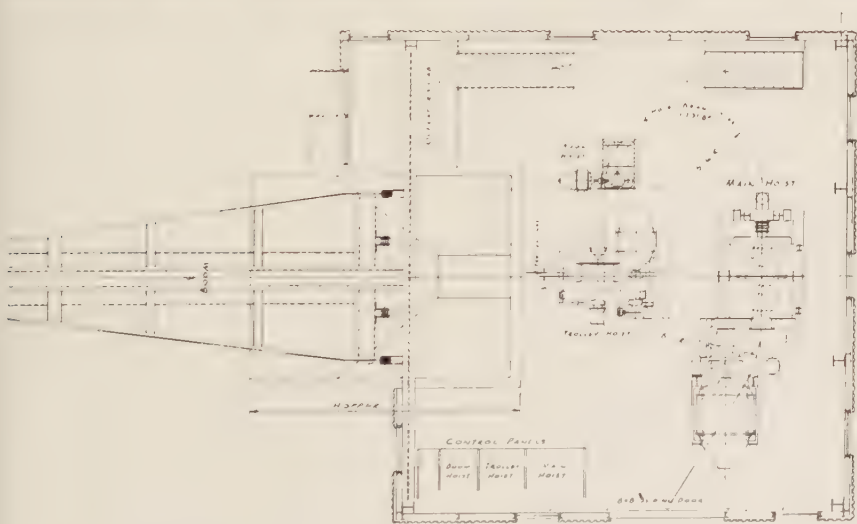


FIG. 5.—TYPICAL ENGINE ROOM ARRANGEMENT ELECTRICALLY OPERATED TOWER

Both the electrically operated and the steam operated towers have a two-rope grab bucket, one line being called the holding line and the other the closing line. The closing line closes the bucket or permits it to open. The holding line holds the bucket as it opens. The two ropes operate in unison from the twin drums of the main hoist when lifting the closed bucket.

The trolley hoist operates the trolley line and controls the inward and outward movement of the bucket. The horizontal component of the pull on the main hoist lines causes the trolley and bucket to move in toward the receiving hopper. A skillful operator hoists and trolleys simultaneously, starting the bucket inward toward the hopper soon after it has cleared the hatches. The path of the bucket approximates a portion of a parabola. Although details of operating controls in towers vary, in general the operation is as follows in the case of an electrically operated tower.

The operator stands with the master controller of the main hoist motor in his right hand and the controller for the trolley motor in his left. With the bucket open in the cargo, the hoist controller is thrown forward, the closing line winds in, closing the bucket jaws into the material. The friction connecting the hoisting drum to its shaft is not sufficient to lift the loaded bucket after it closes but by then a time limit relay which controls the friction of the holding drum operates to throw power onto the holding line. The two ropes lift the load together. The bucket rises and moves inward against dynamic braking of the trolley motor or against the trolley brake and when the bucket reaches dumping position, a brake locks the holding drum. At the same time the master controller is momentarily reversed, releasing the closing line. *The full weight of the bucket now comes on the holding line*, and the bucket therefore swings open, dumping its load into the hopper. It is at the moment of dumping that the greatest load comes on the main hoist cable, one line for an instant carries the full weight of the bucket and its load. The control motor is now reversed, the open bucket is pulled outward and descends under the control of dynamic braking in the main motor or by main hoist frictions and drops into the cargo, completing one full cycle.

Fig. 6 shows a typical analysis of one complete cycle for an electrically operated tower based on 970 ft. per minute rope speed. In this case dynamic braking is used in both the trolley motor and main hoist to assist in braking.

When material taken from the ship is to be deposited in a storage pile back of the tower, a so-called through tower is sometimes used. The grab is suspended in the bight of the holding and closing lines from a 4-sheave trolley hauled along the boom by a separate hoist. For any given rope speed, the vertical movement of the bucket is

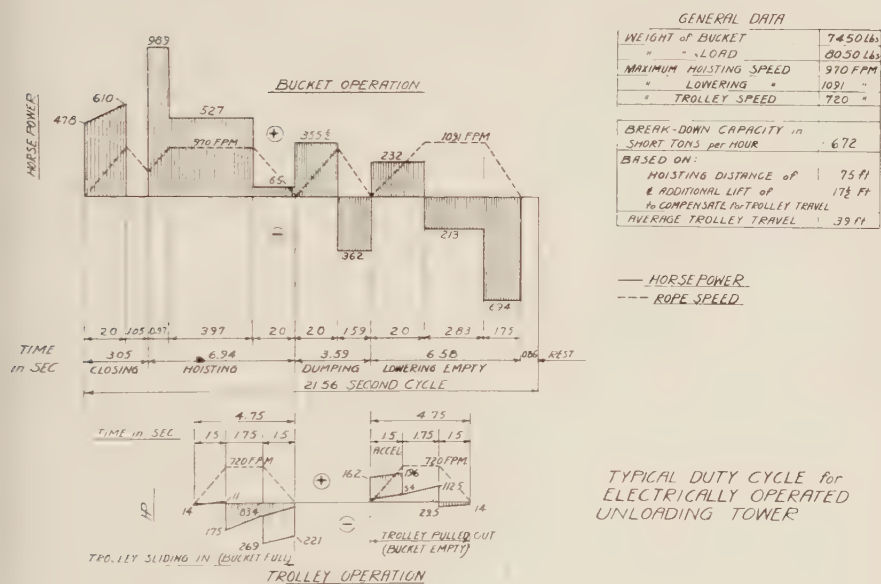


FIG. 6

FIG. 7.—THROUGH TOWER
(Robins Conveyors Inc.)

therefore only half that of the conventional single boom tower. Also since the bucket has to travel a considerable distance horizontally, it makes for relatively slow operation. A typical tower of this type is shown in Fig. 7. To overcome this disadvantage of the through tower, the double boom tower as shown in Fig. 8 has frequently been used. In this case there are two buckets, one on the land side and one on the water side. Coal deposited in the receiving hopper from the ship is carried to the storage pile by means of conveyors. When it is desired to reclaim coal from the storage pile, the water side ropes are disconnected and the land side ropes attached to the hoist drums. This change-over takes about one-half hour.

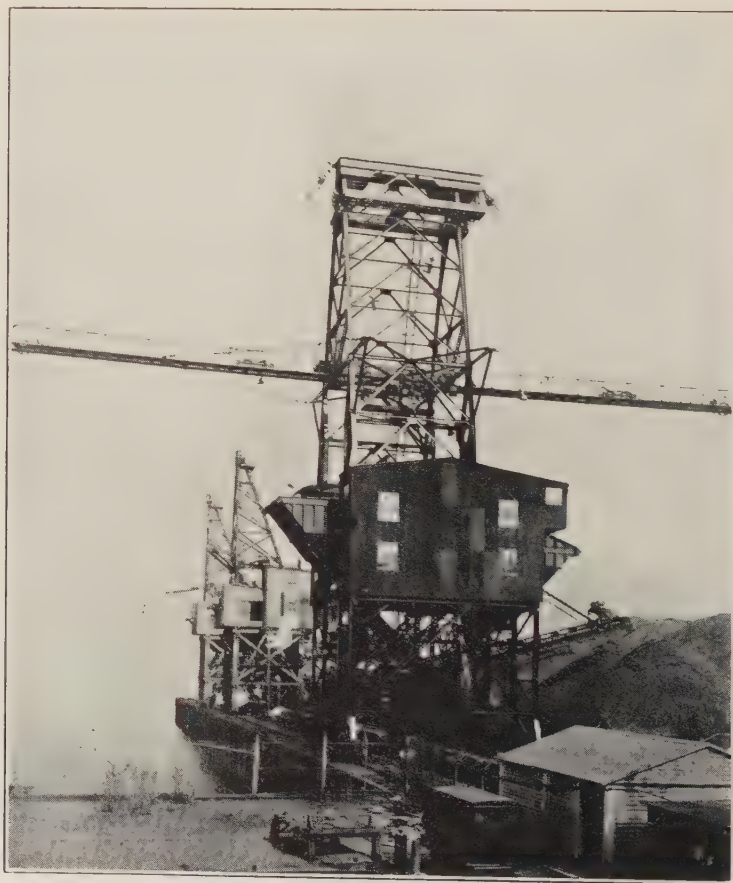


FIG. 8.—DOUBLE BOOM TOWER

Fig. 9 shows a 4-ton travelling electrically operated tower with a stacker boom delivering the coal to a storage pile directly behind the tower.

In the design of an unloading tower, it is necessary to know what type of ship is expected so that proper clearance can be provided.

In the case of towers extending to the wharf level, the front legs are generally kept 4 to 8 ft. back of the fender line. The lip of the hopper should be high enough above the wharf to clear the ship's bridge structure when the ship is light at high tide.

It is desirable to have the boom clear the masts. This is not necessary, however, because the boom can be arranged to be swung clear of the masts or be lifted to clear the masts when the tower or the ship is moved.

The width of the base of the tower is determined by wharf conditions and by requirements of stability.

When the preliminary outline of the tower has been determined



FIG. 9.—ELECTRICALLY OPERATED TOWER WITH 4-TON CAPACITY BUCKET

and the equipment selected, a general loading diagram is made and is studied for three different conditions:

- 1—Maximum operating loads with the trolley at the outer end of the boom at the moment of lifting the bucket and with wind on the back of the tower at 5 to 10 lbs. per sq. ft. of gross area. This condition generally gives minimum load on the rear wheels and maximum load on the front wheels.
- 2—The tower not operating but with full wind load on the rear. This wind load is generally taken at 20 to 30 lbs. per sq. ft. of solid area and with the boom swung in. This may also give maximum loads on the front wheels and minimum load on the rear wheels.
- 3—Tower not operating, the boom swung in and full wind on the face of the tower. This condition gives maximum loading on the rear wheels and minimum loading on the front wheels.

For maximum wheel loading the effect of wind parallel to the tower track should also be considered.

The design of the structure itself must take into account the operating loads, wind loads and dead loads, and in this analysis there is nothing unusual. One hundred percent is usually added to the gross weight of the bucket and its contents to allow for impact effect. These operating loads are the principal loads on the upper frame or, as it is generally called, the A-frame of the tower.

The bucket load passes up through the trolley in the holding and closing lines up to the peak sheaves where the rope turns downward to the engine thereby introducing the principal operating load into the tower.

Another important force in the A-frame is the thrust from the boom caused by the pull on the trolley line due to the horizontal component of the stresses in the operating ropes.

From the engine floor down, there are of course in addition stresses from the operating loads, the effect of the dead load of the engine and floor and other mechanical equipment.

Impact effect from the engine in calculating supporting members is generally taken at 50 percent.

These unloading towers get extreme abuse in operation. It is therefore wise to use low basic stresses. It is considered good practice

to use basic stresses of 16,000 lbs. per sq. in. for design of the principal portions of the structure and in places subject to shock, 12,000 lbs. In spite of the greatest care in designing these structures, local damage will occur due to such things as damage from the bucket when out of control. The receiving hoppers particularly get very severe use. There is no reason why an operator cannot lower the bucket easily into the hopper, but at the end of a long, strenuous day and particularly after dark, he is likely to allow it to drop free 8 or 10 feet into the hopper with resulting damage.

It is a good plan to use liner plates in the receiving hoppers and



FIG. 10.—STEAM OPERATED UNLOADING TOWER WITH 4-TON CAPACITY BUCKET

also heavy wearing strips for the bucket to slide on when it is being lowered into the hopper. The hopper lip itself is many times hit by the bucket. A heavy oak timber at this point protects the hopper from excessive damage and at the same time does not damage the bucket itself as heavy steel members at this point would.

Fig. 10 shows a tower which has some interest to structural designers because of the use of timber for the tower base and steel for the A-frame. This tower has a 4-ton grab bucket, and it was not considered good practice to use a timber A-frame because of the high



FIG. 11.—WHIRLEY TYPE TOWER
(Clyde Iron Works)

operating stresses. Timber was used in the base for economy. The required strength in the joints was obtained by use of split ring connectors.

Fig. 11 shows a modern tower of unusual design, utilizing a Whirley type engine for the main hoist and having a pivotted A-frame which is pulled back by the boom hoist when it is desired to raise the boom. The center of gravity of the boom and A-frame weight is always out beyond the hinge pin and so maintains tension in the ropes leading to the boom hoist.

When a tower is not in operation, it is essential that it be securely locked in position to prevent damage from windstorms. This may be accomplished by lashing the tower to the wharf structure or it may be accomplished by guying the tower to fixed anchorages in the wharf with wire rope or it can be done by the use of automatic rail clamps. These are patented devices which operate to clamp the rail head automatically whenever the tower is not being propelled. This clamping is effected through powerful jaws and is no doubt the safest type of device because these clamps are always in operation except during the brief times when the tower is being shifted from one hatch to the next.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

APRIL 24, 1946.—A Joint Meeting of the Boston Society of Civil Engineers with the Northeastern Section, American Society of Civil Engineers, was held this evening at the Engineers Club. Forty-four members and guests attended the meeting and dinner.

President, Harvey B. Kinnison, of the Northeastern Section, American Society of Civil Engineers presiding at the Joint meeting, called upon President, George A. Sampson, of the Boston Society of Civil Engineers to conduct any BSCE matters of business.

President Kinnison then introduced the speaker of the evening, Senator Arthur Coolidge, Chairman of the Rapid Transit Commission, who gave an interesting talk on "The Proposed New Rapid Transit System of Metropolitan Boston".

Adjourned at 9:20 P.M.

EVERETT N. HUTCHINS, *Secretary*

MAY 15, 1946.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the 20th Century Association, 3 Joy Street, Boston, Mass., and was called to order by President, George A. Sampson, at 7:00 P.M. Forty-four members and guests were present and forty-four persons attended the dinner.

President Sampson announced the death of the following members:—

William P. Morse, who was elected a member May 17, 1893, and who died February 21, 1945; George S. Brush, who was elected a member January 24, 1945, and who died April 28, 1946.

Attention was called to a bill now before Congress which would designate the so-called Boulder Dam as the Hoover Dam. The members of the Society present voted that the Secretary of this Society be instructed to notify the proper persons that at a meeting of the Boston Society of Civil Engineers held on May 15, 1946, it was the sentiment of those present that the bill before the Congress should be passed and the name of the dam become Hoover Dam.

President Sampson introduced the speaker of the evening, Mr. William P. Homans, Regional Director for New England of Civilian Production Administration, who gave a very interesting paper on "The Construction Program". A question period following the talk proved of interest also.

A rising vote of thanks was given the speaker.

Adjourned at 9:00 P.M.

EVERETT N. HUTCHINS, *Secretary*

SANITARY SECTION

MARCH 6, 1946.—A meeting of the Sanitary Section was held this evening at the Society Rooms at 7:00 P.M., following an informal dinner gathering at the Ambassador Restaurant. Sixty-

one persons attended the meeting, with forty at the dinner.

Chairman Mellish first called upon President Farwell, who addressed the meeting briefly.

The report of the Executive Committee on the activities of the Section for the past year was then read by the Clerk, accepted and placed on file.

The Chairman then read the report of the Nominating Committee, consisting of G. W. Coffin, Chairman, C. O. Baird, Jr., and W. L. Hyland. The Committee submitted the following nominations for officers for the ensuing year:

Chairman—Walter E. Merrill

Vice-Chairman—Edwin B. Cobb

Clerk—George C. Houser

Executive Committee—George Brousseau, Murray H. Mellish, William E. Stanley

Upon motion duly made and seconded, it was voted that the clerk cast one ballot for the list of nominees submitted by the Nominating Committee.

Chairman Mellish then called upon Mr. Merrill, the newly-elected Chairman, who spoke briefly.

The speaker of the evening was George W. Coffin, Consulting Engineer, Coffin & Richardson, Inc., who presented a very interesting illustrated paper, entitled "Design of Proposed Boston Refuse Incinerator". The speaker discussed the design of an incinerator to be located on Southampton St., and equipped with waste-heat boilers, to generate steam for furnishing heat and power to the City Hospital.

After considerable general discussion, the speaker was accorded a rising vote of thanks and the meeting adjourned at about 9:15 P.M.

GEORGE C. HOUSER, *Clerk*

DESIGNERS' SECTION

APRIL 10, 1946.—Following the usual informal dinner at the Ambassador Restaurant at 6:50 p.m., Chairman Peabody

called the meeting to order in the Society rooms at 715 Tremont Temple. The Clerk's report of the previous meeting of March 13, 1946 was read and approved.

Professor Peabody introduced the speaker of the evening, Mr. J. Stuart Crandall, President and Chief Engineer, Crandall Dry Dock Engineers, who gave a well illustrated paper entitled "Comments on Breakwaters".

Mr. Crandall began his discussion with some slides illustrating the use of breakwaters in forming some of the harbors of the ancient cities of Tyre, Alexandria and Ostia as well as some of the more modern European harbors. A brief discussion of wave action and pressures as it effects the placing and cross-sectional design of breakwaters followed. During the latter part of the talk many slides of typical European breakwaters of the Mound and Vertical type and a few American breakwaters were shown. Some failures of breakwaters were shown and reconstruction plans discussed.

Attendance: 44 members and guests.

ERNEST L. SPENCER, *Clerk*

MAY 8, 1946.—The meeting was called to order by Chairman Peabody at 7:00 P.M. in the Society Rooms at 715 Tremont Temple after an informal dinner gathering at the Ambassador Restaurant.

The Clerk's report of previous meeting was read and approved. The clerk also gave a brief statement concerning the program of speakers that have already agreed to speak to the group beginning in October.

The speaker of the evening, Mr. Abraham Woolf, was then introduced by the chairman. Mr. Woolf had chosen "Factory Roofs" as the topic of his talk. After briefly discussing the growth of the present day manufacturing plant from the old 15-20 ft. bay multiple story building to the present day enormous single floor areas with

bays from 60 feet up to 200 feet in dimensions, the speaker discussed the advantages and disadvantages of roof construction using wood, concrete, and steel.

The various elements of roof construction such as the purlin, member carrying the purlin, carrying truss, column, and bracing were defined and design limitations considered. Throughout this entire part of the discussion, the problem of adequate day-lighting was of prime importance;—this item concerning an area of about 30% of the roof. Mr. Woolf illustrated his talk with numerous slides of modern manufacturing plants from various parts of the country. These slides showed very clearly the various types of roof designs and uses of each type.

After a discussion period, the meeting adjourned at 8:30 P.M. Thirty-six members and guests were present.

ERNEST L. SPENCER, *Clerk*

HIGHWAY SECTION

JANUARY 30, 1946.—A regular meeting of Highway Section was held this evening at Northeastern University at 7:30 P.M., following an informal dinner gathering at the Lobster Claw Restaurant. Twenty-one persons attended the meeting.

The Chairman, William C. Paxton, introduced the speaker of the evening, Mr. Ralph Hansen of the United States Engineering Office, who gave an extremely interesting talk on "The Design of Flexible Pavements." The talk was illustrated with slides, showing the vast data collected by the United States Engineering Office on tests run under actual field conditions. An interesting discussion period followed the talk.

A rising vote of thanks was given to the speaker.

Adjourned at 9:00 P.M.

GEORGE W. HANKINSON, *Clerk*

FEBRUARY 27, 1946. — The Annual

Meeting of the Highway Section of the Boston Society of Civil Engineers was held this evening at 7:30 P.M., at Richards Hall, Northeastern University, following an informal dinner gathering at the Old France Restaurant. Twenty-three persons attended the meeting.

The Chairman, William C. Paxton, read the names of the new officers nominated by the Nominating Committee. The Officers for the coming year are: Chairman, Francis T. McAvoy; Vice-Chairman, George W. Hankinson; Clerk, John F. Shea; Executive Committee, William C. Paxton, John B. Babcock, Thomas C. Coleman. The Secretary cast one vote for the slate of officers.

The speaker of the evening was Mr. Philip Kitfield, Assistant Project Engineer, Massachusetts Department of Public Works, who gave an extremely interesting talk on "Massachusetts Department of Public Works Solution to the Traffic Problems in Boston." An interesting discussion period followed the talk.

Adjourned at 9:30 P.M.

GEORGE W. HANKINSON, *Clerk*

TRANSPORTATION SECTION

(Formerly Highway Section)

APRIL 17, 1946.—The initial meeting of the Transportation Section, Boston Society of Civil Engineers, formerly the Highway Section, renamed as Transportation Section by Board of Government on April 8, 1946, was held this evening at Northeastern University at 7:30 P.M., following a meeting of the Transportation Section, Executive Committee meeting at 5:30 P.M., and an informal dinner at the Child's Old France Restaurant at 6:00 P.M. Forty persons attended the meeting.

The Chairman, Francis T. McAvoy, in his general remarks opening the meeting, stated this was the first meeting under the new title Transportation Section, inasmuch as the change from

the old title Highway Section, had been approved by the Boston Society of Civil Engineers.

The speaker of the evening was A. J. Bone, Assoc. Professor of Highway Engineering, Massachusetts Institute of Technology, whose subject was "Airport Site Planning." The talk covered two sections, A) Selection of Site and B) Layout of Field and Runways. A discussion period followed the talk.

After a rising vote of thanks was given the speaker, the meeting was adjourned at 9:30 P.M.

JOHN F. SHEA, *Clerk*

HYDRAULIC SECTION

FEBRUARY 6, 1946.—A meeting of the Hydraulics Section was held in the Society rooms, 715 Tremont Temple, following a dinner at the Ambassador Restaurant. During a business meeting conducted by Vice-Chairman Harold A. Thomas, Jr., the report of the nominating committee was read. In the ensuing vote the following were elected to serve as officers for the coming year.

Chairman: Harold A. Thomas, Jr.

Vice-Chairman: Leslie J. Hooper

Clerk: John G. W. Thomas

Executive Committee: James F. Brittain, Elliot F. Childs, Gardner K. Wood

The speaker of the evening was Murray H. Mellish, Hydraulic Engineer, New England Division U. E. Engineers Office, who spoke on "Hydraulic Features of a Flood Control Project". Slides showing finished and proposed flood control projects in the Merrimack and Connecticut valleys were also shown.

An unusually lively discussion after the talk was evidence of the interest in the material presented by Mr. Mellish. The meeting adjourned at 8:50 P.M.

Thirty-eight members and guests attended the meeting.

JOHN G. W. THOMAS, *Clerk*

MAY 22, 1946.—A meeting of the Hydraulic Section was held this evening at the Society Rooms, 715 Tremont Temple, Boston, following an informal dinner at the Ambassador Restaurant. Chairman Harold A. Thomas, Jr., introduced the speaker of the evening, Mr. Byron O. McCoy, of Chas. T. Main, Inc., who gave a very interesting and informative talk on the "Hydraulic Design of Intake Structures".

The talk was followed by an open discussion period, with 36 members and guests present.

The meeting adjourned at 8:15 p.m.

JOHN G. W. THOMAS, *Clerk*.

APPLICATIONS FOR MEMBERSHIP

[July 15, 1946]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

Edwin M. Corry, Waban, Mass. (b. September 3, 1880, Summerfield, Louisiana.) Graduated from Louisiana State University, June, 1902, B.S. degree in Civil Engineering. Following a brief period of service with Southern Pacific Railroad on maintenance of way and location surveys, obtained employment with the War Department Engineers in September, 1903. Except for a period of about three years during the first World War, and one year as superintendent of a dredging company, my service with the War Department Engineers has been continuous. The principal engineering works on which I have been engaged are as follows:—1903-1904, Inspector on construction and surveys of levees of the lower Mississippi River; 1905-1917, Engineer on location, construction and maintenance of navigation channels and waterways in southeast Texas and southwest Louisiana. These works involved jetty construction, dredging operations for ocean vessels by government plant and by contract, shore protective works, etc.; 1917-1920, Officer, U. S. Army; 1920-1922, in charge of U. S. Engineer Field Office, San Juan, Porto Rico, with duties in connection with the enlargement of San Juan Harbor and the reclamation of marshy areas adjacent thereto. The duties also included investigations and reports on the enlargement of the harbors of Ponce, Porto Rico, and Christianstad, St. Croix, V. I.; 1922-1925, in local charge of the improvement of Southwest Pass, mouth of Mississippi River, for a navigable depth of 35 feet. The work involved the design and construction of jetties, retaining walls permeable dikes for channel constriction, dredging by means of pipe-line and seagoing hopper dredges; 1925-1926, engineer on river and harbor investigations, reports, and construction works coming under the United States District Engineer Office,

Boston, Mass. 1926-1927, superintendent for Breymann Dredging Company engaged in the supervision of dredging operations in the entrance channel to Boston Harbor the Fore River Channel, Quincy, Mass., and dredging channels for outfall sewers at Lynn, Mass., and Gloucester, Mass.; 1927-1936, in charge of investigations and reports for the U. S. Engineer Office, Duluth, Minn., relative to the settlement of flowage damage claims and the acquisition of flowage easements as provided for by treaty between the United States and Great Britain, which treaty prescribed the means and method of disposing of boundary problems growing out of the raising of the level of Lake of the Woods by Canadian water power interests. The engineering duties in the Duluth District Engineer Office also included dredging operations and the design and construction of breakwaters, dikes and other harbor protective works; 1936-1942, engineer in the United States District Engineer Office, Boston, Mass., in charge of investigations and reports for river and harbor improvements, the acquisition of lands or rights in lands for river and harbor and flood control works, investigations and reports on damage claims, and the review of plans and specifications for river and harbor and flood control works; 1942 to date in charge of the engineering division of the U. S. Division Engineer Office, New England Division, Boston, Mass., with general supervision of the review of plans, specifications, estimates, and operations of the U. S. District Engineer Office of Boston, Mass., and Providence, R. I., in connection with construction works for the Army and for river and harbor and flood control Works. Refers to *H. B. Kinnison, C. R. Gow, M. H. Melish, H. I. Wyner.*

Jack E. McKee, Stoneham, Mass. (b. November 9, 1914, Pittsburgh,

Penna.) Carnegie Technology, B.S. in Civil Engineering, 1936; Harvard University, S.M. in Sanitary Engineering, 1939; Harvard University, D.Sc. in Sanitary Engineering, 1941. Experience, employed by Tennessee Valley Authority from June, 1936 to September, 1937, in Hydraulic Data Division. Resigned to accept assistantship at Harvard University. Completed duties and degrees in June, 1941, and accepted a position with the U. S. Public Health Service as Associate Public Health Engineer, in New Orleans District office. Called to active duty in Army, Corps of Engineers in April, 1942, as officer in charge of the Sanitary Engineering Section, Repairs and Utilities Branch of the Division Engineers Office, Salt Lake City. Entered School of Military Government in Charlottesville, Virginia and went overseas with First Army as Sanitary Engineer for Military Government in combat areas. With First Army from Normandy to VE Day. Released from active duty (as a Major, CE) on January 1, 1946. Started to work as assistant engineer for Thomas R. Camp on January 2, 1946. At present engaged in consulting engineering at 6 Beacon Street, Boston, Mass. Refers to *T. R. Camp, H. G. Dresser, H. M. Turner, G. M. Fair, H. A. Thomas, Jr.*

WILLIAM C. MOBERGER, Everett, Mass. (b. June 20, 1911, Everett, Mass.) Graduated from Northeastern University in June 1934, B.S. degree in Civil Engineering. Cooperative work with Aspinwall & Lincoln and McDonough Construction Company. July 1934-June 1936, Civil Engineering with New England Survey Service Inc.; July 1936-September 1942, Manager of Philadelphia Survey Service, Philadelphia, Pa.; 1943-1946 Lieutenant U. S. Navy. At present Sales Manager and Civil Engineer of the New England Survey Service Inc. Engineer in charge of Field Layout for New England Survey

Service at Devens, Presque Isle and Houlton Air Bases. Refers to *C. O. Baird, L. A. Chase, A. E. Everett, E. A. Gramstorff.*

Transfer from Grade of Junior

Chesley F. Garland, Jacksonville, Florida. (b. April 14, 1916, Cambridge, Mass.) Graduated from Northeastern University in 1938, B.S. degree in Civil Engineering; M. S. in Sanitary Engineering, Harvard, 1939. Biostatistics work at Johns Hopkins (40-41). Experience, June-October, 1939, field construction parties for Metropolitan District Water Supply Commission, Boston, Mass.; October, 1939-April 1941, junior assistant Sanitary Engineer Maryland State Board of Health, reviewing operation of public water and sewage and industrial wastes treatment works; April, 1941-June, 1942, assistant sanitary engineer, Florida State Board of Health; June, 1942-May, 1946, 1st Lt. to Major, SnC., Army. Duty as Sanitary Engineer in U. S. posts, Office of the Surgeon General and two years in European Theatre of Operations with Allied Military Government with final assignment as Military Government Sanitary Engineer for Land of Bavaria, Germany, supervising sanitary engineering activities of German Civil Government for an area having 10 million inhabitants; May, 1946 to date, Sanitary Engineer with Florida State Board of Health. In charge of water supply and wastes disposal work of Bureau of Engineering of the State Department for State of Florida. Includes review of plans and specifications of proposed projects, review of plant operation and recommendations for new or proposed municipal and for industrial wastes and water treatment works. Refers to, *C. O. Baird, G. M. Fair, E. A. Gramstorff, R. Soule, H. A. Thomas.*

Richard Halloran, Newton Highlands, Mass. (b. February 5, 1914, Boston,

Mass. Graduated from Massachusetts Institute of Technology, course in Building Engineering and Construction. Experience, July, 1936 to June, 1936 with various employers in land surveying and as assistant and superintendent of construction on private houses and U. S. Post Offices; June, 1938 entered family firm of Edward Halloran, Inc., engaged in landscape planting and construction; May, 1942, entered U. S. Navy and U. S. Naval Academy, Annapolis, Md., and was commissioned as Ensign USNR in September, 1942; September, 1942 to March, 1945 served on board USS Kearny (DD432) becoming engineer officer in November, 1943; April, 1945 to December, 1945 served in fitting out and later on board as engineer officer on USS HAROLD J. ELLISON (DD864). Served short time as executive officer on this vessel. January, 1946 was detached from active duty with rank of Lieut. USNR and am now engaged as treasurer, Edward Halloran, Inc., Newton Highlands, Mass. Refers to *A. G. Dietz, H. G. Protze, W. C. Voss.*

HYMAN B. ULLIAN, Newton, Mass. (b. August 7, 1895, Lowell, Mass.) Graduated from Mechanic Arts High School, Boston, Mass., in 1912; Massachusetts Institute of Technology in 1916. Experiences, June 1916-December 1916, inspector and Chief of Party on Highway Construction for the Massachusetts Highway Commission; December 1916-April 1917, designed part of structure of Springfield Bridge for Fay, Spofford & Thorndike, Boston, Mass.; April 1917-June 1917, engineer in charge of layout of roadway system at Camp Custer for United States; June 1917-November 1917, designed layout of streets, water, sewer and paving of Langley Field, Virginia; December 1917-May 1918, designed layout of underground structures on Boston Army Base, Boston, for Fay, Spofford &

Thorndike; May 1918-February 1919 engineer U. S. Navy in charge of layout and construction of Harvard Radio Barracks and Navy Coal trestle at South Boston; February 1919 to present, private practice in civil engineering and surveying. Refers to *C. O. Baird, C. B. Breed, A. E. Everett, L. A. Chase.*

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J. DONALD MITSCH, January 18, 1946
JOSEPH R. HEWS, January 28, 1946
THEODORE R. KENDALL, February 4,
1946
RALPH WHITMAN, February 3, 1946
WILLIAM P. MORSE, February 21, 1946
GEORGE S. BRUSH, April 28, 1946

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INDEX TO ADVERTISERS

	PAGE
ALGONQUIN ENGRAVING CO., INC., 18 Kingston St., Boston	vii
BAILEY, HOWARD E., 6 Beacon St., Boston	v
BARBOUR, FRANK A., Tremont Building, Boston	iv
BARROWS, H. K., 6 Beacon St., Boston	iv
BUFF & BUFF INSTRUMENT CO., Jamaica Plain	vii
BUILDERS, 84 Coddling St., Providence 1. R. I.	vii
CAMP, THOMAS R., 6 Beacon Street, Boston	iv
CHAPMAN VALVE MFG. CO., 165 Congress St.	vi
CRANDALL DRY DOCK ENGINEERS, 238 Main St., Cambridge	v
CROCKER, WILLIAM S., 46 Cornhill, Boston	iv
ELLIS, W. H., & SON CO., East Boston	vi
ENGINEERING ASSOCIATES, 146 Summer St., Boston	vi
FAY, SPOFFORD & THORNDIKE, 11 Beacon St., Boston	iv
GAHAGAN CONSTRUCTION CORP., 90 Broad Street, New York, N. Y.	viii
GANTEAUME & MCMULLEN, 99 Chauncy St., Boston	v
GOULD, GARDNER S., 89 Broad St., Boston	v
GOW COMPANY, INC., 956 Park Square Building, Boston	v
HAWKRIDGE BROS., 303 Congress St., Boston	vii
HEFFERNAN PRESS, 150 Fremont St., Worcester	viii
HOLZER, U., INC., 29 Collins Street, Hyde Park	vii
HUGHES, EDWARD F., COMPANY, 53 State St., Boston	vi
JACKSON & MORELAND, Park Square Building, Boston	iv
LINENTHAL, MARK, 16 Lincoln St., Boston	v

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	PAGE
MAIN, CHAS. T., INC., 21 Devonshire St., Boston	iv
MAKEPEACE, B. L., INC., 387 Washington St., Boston	Back cover
MC CREERY AND THERIAULT, 131 Clarendon St., Boston	vi
METCALF & EDDY, Statler Building, Boston	iv
MULCARE, THOMAS, CORP., 66 Western Ave., Boston	vii
NEW ENGLAND CONCRETE PIPE CORP., Newton Upper Falls, Mass.	vi
NEW ENGLAND POWER SERVICE COMPANY, 441 Stuart St., Boston	v
NORTHERN STEEL COMPANY, 44 School St., Boston	vi
O'CONNOR, THOMAS, & Co., 238 Main St., Cambridge	vi
OLD CORNER BOOK STORE, THE, 50 Bromfield St., Boston	vii
PIPE FOUNDERS SALES CORP., 6 Beacon Street, Boston	vi
REIDY, MAURICE A., 101 Tremont Street, Boston	v
S. MORGAN SMITH CO., 176 Federal St., Boston	vi
SPAULDING-MOSS CO., Boston	vii
STONE & WEBSTER ENGINEERING CORP., 49 Federal St., Boston	v
STUART, T., & SON COMPANY, 70 Phillips St., Watertown	vi
THOMPSON & LICHTNER CO., INC., THE, 8 Alton Place, Brookline 46, Mass.	v
TURNER, HOWARD M., 6 Beacon St., Boston	iv
WARREN FOUNDRY & PIPE COMPANY, 11 Broadway, N. Y.	vii
WESTON & SAMPSON, 14 Beacon St., Boston	iv
WHITMAN & HOWARD, 89 Broad St., Boston	iv
WORCESTER, J. R., & Co., 79 Milk St., Boston	iv
WORTHINGTON, E., 26 Norfolk St., Dedham	v

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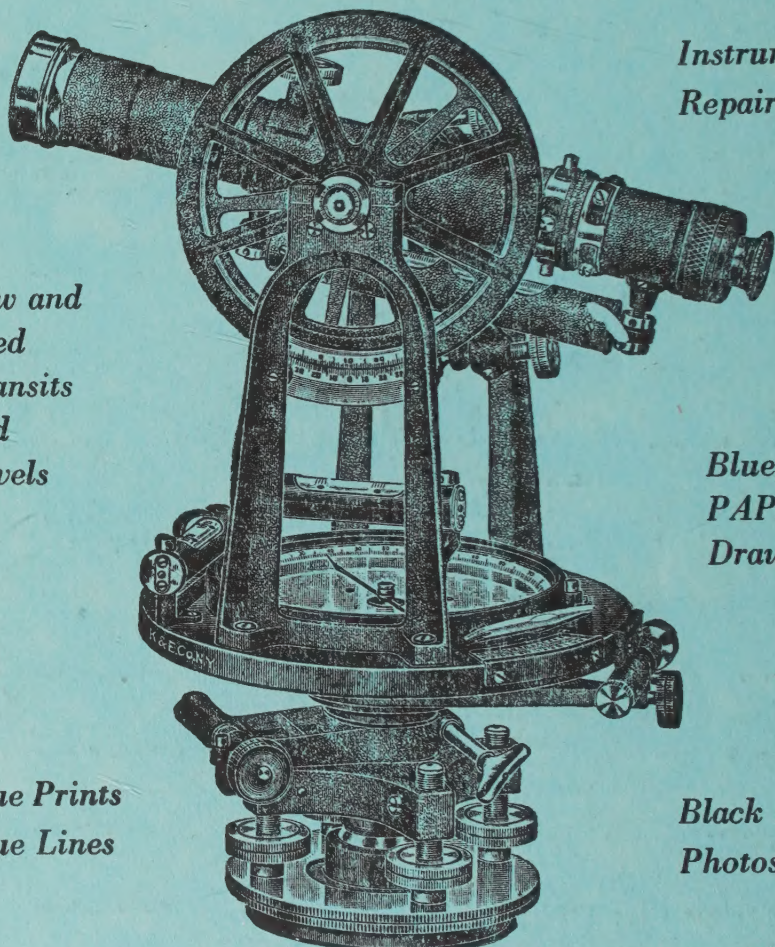
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